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EVALUATION OF DIFFERENT COMPUTATIONAL MODELLING STRATEGIES FOR THE ANALYSIS OF LOW STRENGTH MASONRY STRUCTURES

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ABSTRACT

Masonry is a composite material characterized by a large variability of its constituent materials. The materials used, the quality of the bond and variations in the standard of workmanship significantly affect the mechanical performance of the overall masonry structure. Masonry structures, especially the historical ones, are usually characterized by low strength, due to a variety of reasons, namely low units and/or mortar strength or low bond; this makes more difficult to study these types of structures according to general rules because of different structural schemes. The aim of this paper is to evaluate the suitability of continuous FEM (Finite Element Method) or DEM (Distinct Element Method) approaches to analyse the behaviour of low strength masonry and to contribute to the knowledge and selection of the best approach with a cost and time effective solution. The comparison with experimental results on different low strength masonry validated the approaches and showed that, for low bond strength masonry, DEM approaches performed better compared to low unit strength masonry where the emphasis on joint behaviour in DEM approaches is less effective because the weak component is the unit.

Keywords: Masonry modelling, low strength masonry, finite element analysis, distinct element analysis.

1 INTRODUCTION

Masonry is the generic term for a composite material made of a large number of separate small elements (units) bonded together by some binding filler (mortar) in many very different arrangements. The materials used, the quality of the bond and workmanship and the masonry textures significantly affect the mechanical performance of the overall masonry structure.

Masonry structures, especially the historical ones, are usually characterized by low strength, due to a variety of reasons, and mainly these different types of low strength masonry can be outlined:

a) Low bond strength masonry;

b) Low unit strength masonry;

c) Low unit and mortar strength masonry.

Low bond strength masonry **refers to masonry** in which the bond at the unit/mortar interface is such low so that it will have a dominant effect on the mechanical behaviour such as the formation of cracks and the formation of the collapse mechanism. Such type of masonry is encountered in historic constructions where lime mortar were mainly used; masonry arch bridges; tunnels linings and earth retaining walls where unit/mortar joint bond has been disrupted by the action of water leeching through the masonry; and in more recent examples of masonry construction due to lack of quality control on site.

Low unit strength masonry **refers to** masonry in which the strength of the unit blocks has a dominant effect on the mechanical behaviour and failure mechanism. Such type of masonry is encountered in constructions made of tuff blocks. Tuff is a building material used in wall constructions around the world since ancient times. Tuff is characterised as soft, porous rock formed by the compaction and cementation of volcanic ash. Such type of structures is often encountered in Italy, Turkey and Japan.

Low unit and mortar strength masonry **referes to**masonry in which the strength of the units is comparable to the strength of the mortar. Therefore, both the unit and the mortar strength will have a dominant effect on the mechanical behaviour and failure mode. Such type of masonry is encountered in adobe constructions.

The need to predict the in-service behaviour and load carrying capacity of masonry structures has led researchers to develop several numerical methods and computational tools which are characterized by their different levels of complexity. For a numerical model to adequately represent the behaviour of a real structure, both the constitutive model and the input material properties must be selected carefully by the modeller to take into account the variation of masonry properties and the range of stress state types that exist in masonry structures. A broad range of numerical methods is available today ranging from the classical plastic solution methods [1] to the most advanced non-linear computational formulations (e.g. finite element and distinct element methods of analysis). The selection of the most appropriate method to use depends on, among other factors, the structure under analysis; the level of accuracy and simplicity desired; the knowledge of the input properties in the model and the experimental data available; the amount of financial resources; time requirements and the experience of the modeller [2]. It should also be expected that different methods should lead to different results depending on the adequacy of the approach and the information available. Preferably, the approach selected to model masonry should provide the desired information in a reliable manner within an acceptable degree of accuracy and with least cost.

However, the selection of a suitable method of analysis is not an easy task. Several comparative studies to identify the capabilities and limitations of each method of analysis have been carried out in the past [3,4,5,6,7]. Such studies are mainly focused on comparing the load displacement results of the large scale experiments against those obtained from the different computational model. However, none of these studies investigated the suitability of the method to different types of masonry.

The aim of this paper is to evaluate the suitability of different modelling approaches for the analysis of two different types of masonry by comparing the numerical results with the experimental data obtained. The low strength masonry constructions investigated are: a) a low bond strength brick masonry wall panel with opening and b) a low unit strength masonry wall constructed with tuff. Analysis is being carried out using the computational software DIANA for the application of the finite element method with continuous elements and the software UDEC for the distinct element modelling. Comparisons are made in respect to the suitability of the software to predict the development of the crack patterns under incremental loading; the load at first visible cracking; the failure load; the failure mechanism and the load against deflection relationship.

2 MODELLING APPROACHES FOR MASONRY

Masonry structures are made up of several assemblages of constituent materials. This large variability results in a very difficult definition of limited and specific structural and damage analysis techniques for masonry structures. Although refined Finite Element models or Distinct Element models can be profitably employed to investigate the mechanical behaviour of masonry structures through different numerical strategies. However their use in prediction analyses is still critical as they require high computational effort and expert engineering judgment in the interpretation of numerical results. A significant progress has been attained in the last years about the possibility of performing linear and non-linear approaches that can be carried out according to different levels of detail. Because of affordable non-linear FEM analyses applied to continuum require high expertise, several methods based on distinct elements have been developed too.

2.1 Overview of modelling masonry with FEM

To perform a FEM analysis on masonry structures it is possible to use different modelling approaches. These include equivalent frame [8,9,10], equivalent material approach [11,12] and micro-modelling [13,14]. The equivalent frame approach is typically used to study the in plane behaviour of masonry structures containing openings or entire structures under vertical and horizontal forces. In this approach each wall with openings is meshed as a two dimensional frame by extending of the contour lines of the openings into “pier panels”, “spandrel panels” and “joint panels” which are respectively vertical, horizontal and jointing components. In the equivalent material approach also known as “macro element approach” the masonry is modelled as a homogeneous material achieving equivalent mechanical properties using homogenization techniques. The micro-modelling approach introduced for the first time by Page [15], bricks and mortar are modelled separately. This approach make possible to use different mechanical parameters, different constitutive laws and to allow for local failure of the bricks and the mortar. Furthermore. it is possible to model the mortar bed with frictional interfaces [16] or without frictional interfaces according to the smeared cracking approach [17].

2.2 Overview of modelling masonry with DEM

According to Lemos [18], several numerical modelling techniques (e.g. DDA, YADE, EDEM, BALL, DEM) are based on the Discrete Element Method (DEM). However, a numerical code falls into the category of Distinct Element Method only if:

- It allows finite displacements and rotations of distinct bodies, including complete detachment;

- It recognizes new contacts automatically as the calculation progresses.

Without the first attribute, a numerical code cannot reproduce some important mechanisms in a discontinuous medium; without the second, the numerical code is limited to small numbers of bodies for which the interactions are known in advance. There are four main classes of numerical codes that conform to the proposed definition of Discrete Element Method:

- Distinct Element codes: these programs use explicit time-marching to solve the equations of motion directly. Bodies may be rigid or deformable; contacts are deformable.
- Modal Method codes: the method is similar to the distinct element method in the case of rigid bodies but, for deformable bodies, modal superposition is used.
- Discontinuous Deformation Analysis codes: contacts are rigid, and bodies may be rigid or deformable. The condition of no-interpenetration is achieved by an iteration scheme; the body deformability comes from superposition of strain modes.
- Momentum-Exchange Method codes: Both the contacts and the bodies are rigid: momentum is exchanged between two contacting bodies during an instantaneous collision. Frictional sliding can be represented.

The term Distinct Element Method (DEM) was coined by Cundall [19] to refer to the particular DE scheme that uses deformable contacts and an explicit, time-domain solution of the original equations of motion (not the transformed, modal equations). In particular, such method was originally used in rock engineering projects where continuity between the separate blocks of rock does not exist. The software UDEC falls into the category of Distinct Element. Recently, DEM modelling has also been used for masonry structures. Typical examples of masonry structures that have been modelled using UDEC are described by [6,19,20]. In the distinct element method masonry bricks or blocks are represented as an assembly of rigid or deformable blocks which may take any arbitrary geometry. Rigid blocks do not change their geometry as a result of any applied loading. Deformable blocks are internally discretised into finite difference triangular zones. These zones are continuum elements as they occur in the finite element method (FEM). However, unlike FEM, in the distinct element method a compatible finite element mesh between the blocks and the joints is not required. Mortar joints are represented as zero thickness interfaces between the blocks. Representation of the contact between blocks is not based on joint elements, as occurs in the continuum finite element models. Instead the contact is represented by a set of point contacts with no attempt to obtain a continuous stress distribution through the contact surface. The assignment of contacts allows the interface constitutive relations to be formulated in terms of the stresses and relative displacements across the joint. The unknowns are the nodal displacements of the blocks. However, unlike FEM, the unknowns in the distinct element method are solved explicitly by differential equations from the known displacement while Newton’s second law of motion gives the motion of the blocks resulting from known forces acting on them. So, large displacements and rotations of the blocks are allowed with the sequential contact detection and update of tasks automatically. This differs from FEM where the method is not readily capable of updating the contact size or creating new contacts. This method is also applicable for quasi-static problems using artificial viscous damping controlled by an adaptive algorithm.

3. LOW BOND STRENGTH MASONRY WALL PANELS WITH OPENINGS

Four single leaf unreinforced masonry wall panels (S1, S2, S3 & S4) were tested in the laboratory. The wall panels were developed to represent the clay brickwork outer leaf of an external cavity wall containing openings for windows. All panels were built with a soldier course immediately above the opening with the remainder of the brickwork being constructed in stretcher bond. All wall panels had an opening of 2.025 m (see Fig. 1). The bricks were UK standard size (215 mm × 102.5 mm × 65 mm) Ibstock Artbury Red Multi Stock with a water absorption of 14% and a sand faced finish. The joints were all 10 mm thick, 1:12 (opc: sand) weigh-batched mortar. The bricks and mortar were selected to produce brickwork with a low bond strength, the aim being to represent low quality, high volume wall construction which, in the authors' experience, is fairly typical of low rise domestic construction in the UK. Each panel was constructed on the rigid concrete laboratory floor. As a result the bottom edge of each panel was rigidly supported both in horizontal and vertical direction and the vertical edges were left free. Each wall panel was subjected to a single vertical point load applied at the top of the wall at midspan. The point load was distributed through a steel spreader plate. The load was applied to each wall incrementally. The midspan deflection was recorded at each load increment and each wall was inspected visually for signs of cracking throughout the test. Deflections at ultimate load were not taken for safety reasons and to avoid damage to the dial gauge. The test results are summarised in Table 1.

3.1 Modelling with UDEC

Geometric models representing the clay brick wall/beam panels tested in the laboratory were created in UDEC. Each brick was represented by a deformable block separated by zero thickness interfaces at each mortar joint. To allow for the 10mm thick mortar joints in the real wall panels, each deformable block was based on the nominal brick size increased by 5mm in each face direction to give a UDEC block size of 225 × 112.5 × 75mm. Each block was internally discretised by UDEC into finite-difference zone elements (Fig. 2), each assumed to behave in a linearly elastic manner. In practice, the stresses in the bricks would be well below their strength limit and so no significant deformation would be expected to occur in them.

The mortar joints were represented by interfaces modelled using UDEC's elastic-perfectly plastic coulomb slip-joint area contact option [21]. This provides a linear representation of the mortar joint stiffness and yield limit and it is based upon six parameters namely: normal stiffness of the joint (JKn); shear stiffness of the joint (JKs); joint friction angle (Jfric); joint cohesive strength (Jcoh); joint tensile strength (Jten); and joint dilation angle (Jdil). The material parameters for the masonry constitutive model were obtained from [26] and presented in Table 2.

The bottom edges of the UDEC wall panel were modelled as rigid supports in the vertical and horizontal direction whilst the vertical edges of the wall panel were left free. Self-weight effects were assigned as gravitational load. Initially, the model was brought into a state of equilibrium under its own self weight and then an externally applied load assigned. Histories of mid-span displacement were recorded and a load against displacement relationship was determined (Fig. 3). The little peaks in the curve shown in Fig. 3 represent relaxation of the loading and moment redistribution in the panel due to the formation of a new crack. When a crack propagates there is an abrupt loss of stiffness in the panel. Figs. 4 and 5 show respectively the failure mode of the masonry wall panel predicted with UDEC and observed experimentally. Despite the great variability of masonry [22, 23], good correlation was obtained between the results from the UDEC model and those obtained from the tests in the laboratory.

3.2 Modelling with FEM

The FEM analysis of the low bond strength masonry wall panels was performed in 2D using the software DIANA developed by TNO DIANA [28]. The interaction between mortar joints and brick units modelled using the detailed micro-modelling approach [16]. The geometry of the experimental tests was reproduced modelling mortar and bricks individually without interface elements between them. In Fig. 6 the geometry of the model adopted in DIANA is shown.

The general approach, the selection of element types and material cracking and plasticity models were already successfully employed in previous studies [14,29,30] and they are replicated herein. Interface elements were not considered between mortar and bricks, mainly because reliable experimental mechanical properties of interfaces are not available for this case study. Cracking and plasticity behaviour is provided by combined nonlinear behaviour of mortar and bricks. A regular and dense discretization was used [31] based on the CQ16M eight-node quadrilateral isoparametric plane stress elements with an average dimension of 10 mm have been used for the meshing of both the mortar and the bricks (according to previous studies [14,29,30]. These elements are based on interpolation and Gauss integration [28]. Boundary conditions reproduced the experimental setup. The base sections of the piers of the wall were fixed and the load was applied by means of an imposed displacement by means of a loading platen reproducing the steel platen used in the experimental activity. In Fig. 7 the adopted fine mesh is showed. The main causes of non-linear behaviour of brick masonry are usually non-linear deformation of the bricks and local crack in the masonry [32,33] hence both these effects should be considered in the modelling. The elastic in plane behaviour of both the mortar and the bricks was defined by means of Young Modulus, E, and Shear Modulus, G, while the post elastic in plane behaviour was defined by the multidirectional fixed crack model. In particular Rankine yield criteria in tension and Von Mises yield criterion in compression were adopted. The multidirectional fixed crack model is based on fracture energy. In particular linear softening model in both tension and compression were adopted (Fig. 8). The linear softening curve, which is the simpler softening model, was chosen because the lack of experimental data and because the overall non-linear behaviour of masonry is not strongly conditioned by the deformation characteristics of its components [32,33,34]. This softening model is defined by means of two characteristic values: the strain at the maximum compressive, f_c, (and tensile, f_t, similarly) stress and the ultimate strain (reached when the material is completely softened). The softening behaviour is related to the fracture energy to the equivalent crack bandwidth (this value is automatically computed by the Software [28]. Tensile and compressive strength, fracture energy in compression, G_c, and in tension, G_t, were calibrated by means of the global experimental force/ deflection curve and sensitivity analyses for both mortar and bricks. Except for the Poisson ratio, which is assumed equal to 0.15 for all the materials [30], in Table 3, all the used material parameters are reported. Numerical analyses were carried out under displacement control measuring in plane forces and the smeared crack pattern evolution. The results of the analyses were compared to the experimental outcomes in terms of force-deflection curve and crack pattern. As shown in Fig. 9, the theoretical curve up to about 0.5 mm, is predicted satisfactorily by numerical simulation and the theoretical crack pattern also is close to the experimental as well. On the other hand, the theoretical curve doesn’t simulate the post peak behaviour of the experimental tested panels. In particular the scatter between the theoretical failure point and the experimental failure points is, probably, due to the brittle collapse adopted model. Cracking yields to a fast redistribution of tensile stresses in the cracked areas, and at increasing displacement cracking spreads, yielding to premature failure of the panel. Theoretical and experimental tests mainly showed the same crack pattern (see Fig. 10). The first crack always occurs in the vertical joint in the lower part of the span because of the low bond strength of the vertical joints. It is worth noting that plastic yielding did not occur in the bricks. In the following Table 4 is a comparison between experimental and theoretical results with DIANA in terms of first crack load-first crack deflection is reported.

4 LOW UNIT STRENGTH MASONRY WALL PANELS

Four as built panels were tested in the laboratory under displacement control in order to measure in-plane deformations and strength properties, including the post peak softening behaviour of the specimens. The test setup followed a modified version of ASTM [35], accounting for the dimensions of tuff blocks. Two steel loading supports were placed on the two diagonally opposite corners of the panels to avoid a premature splitting failure of panel edges. All the panels were subjected to diagonal compressive loads forming a 45° angle with the direction of the mortar bed joints (compressive edge load) transferred to the specimen by means of spherical hinge acting in the plane of wall. The panels were built with the global size 1030×1030 mm^2 (aspect height-to-length ratio equal to 1) and bricks size 400 × 110 × 250 mm^3 Masonry units were overlapped on alternate courses and the mortar joint layer dimension was about 15 mm in thickness and less than 250 mm in width as shown in Fig. 11. Tuff bricks were pre-wetted before to build the panel in order to prevent the mortar drying out due to the water absorption of tuff, resulting in poor bond. The used mortar mixture was designed to reproduce typical mechanical properties of mortars used for old tuff masonry buildings. Two LVDTs placed along the diagonals were used to survey the shear deformation over a gauge length of 400 mm. Table 5 shows the main test results. The crack pattern for all the reference tested panels shows a development of initial cracks along the diagonal mortar joints starting at the middle of the diagonal of the wall. The diagonal cracks involve both mortar and bricks; they opened along the compression strut. The workmanship defects can have a big influence on the global response, indeed, for the panel P2, the failure was due to a combination of tensile failure of mortar joints and tuff units (as shown in the Fig. 12a) while in the other cases (i.e. panel P4) the cracks follow a single line of least resistance mainly through the diagonal mortar joints (as shown in Fig. 12b). A full description of the experimental diagonal compression tests on tuff masonry panels is reported by [36].

4.1 Modelling with DIANA

In the case of tuff masonry the weakness of the tuff bricks makes possible the propagation of the crack all over the masonry panel even involving the bricks, so a model able to simulate possible crack in the brick is needed (i.e. is not possible to model the brick as rigid block). The approach adopted for the FEM modelling was the Micro-modelling. Accurate FEM two dimensional numerical analyses have been conducted under plane-stress assumption by means of the TNO DIANA v9.1 code. The panel was modelled by eight-node quadrilateral isoparametric plane stress element based on quadratic interpolation and Gauss integration (see Fig. 13) while the two steel supports were modelled by means of three-node triangular elements. Bricks and mortar are modelled individually, based on exactly the same approach used in the previous case of low bond strength masonry wall panels with openings. The material parameters involved in the numerical simulation are reported in Table 6. Except for both the tensile strength and the Poisson ratio, the parameters are obtained as the average of the values achieved in the experimental tests [36]. The tensile strength has been computed dividing the flexural strength values by 1.2, and the Poisson ratio has been assumed equal to 0.15 for all the materials. Numerical analyses were carried out under displacement control measuring in-plane deformations and stress evolution applying the load through the steel devices according to experimental tests. A uniform probability of defects along the mortar joints has been assumed. Therefore, the workmanship defects (i.e. mortar joints not uniformly and not fully filled) have been simulated by modelling an equivalent reduction of the width (out of plane) of the mortar joints. A numerical test matrix with the mortar joints width considered is reported in Table 7.

The results of the analyses were compared to the experimental outcomes in terms of shear stress against average diagonal strains, and shear stress against average shear strain curves. According to ASTM [35] standard method, the shear stress, τ, has been computed as $\tau = 0.707\ V/A_n$, were V = diagonal load and A_n = net section area of the uncracked section of the panel (in considered case $A_n = 0.092\ m^2$). The average vertical and horizontal strains, ε_v and ε_h have been computed as the average displacement along the compressive and tensile diagonals, respectively, over the same gauge length (400 mm). The shear strain, γ, according to [35], is $\gamma = \varepsilon_v + \varepsilon_h$. The Shear modulus, G, and the Poisson ratio, ν, were computed according to the well-known solid mechanics relationship, as $\nu = -\varepsilon_h/\varepsilon_v$ and $G = \tau/\gamma$ respectively, where E is the Young Modulus. The numerical analyses, in terms of shear strength against average shear strain, fit the experimental results. In particular the smaller considered mortar filling matches the experimental behaviour of the panel P1 (in this case it was argued that the panel P1 had worse behaviour due to the workmanship defects and variability of mortar geometrical properties) and both the fully-filled and half-filled mortar joints analyses match the behaviour of the other as-built panels. A comparison between the numerical and experimental outcomes is plotted in Fig. 14. The partial filling or reduced width of the mortar joints used to include workmanship defects well simulates the experimental results. This result becomes evident comparing the experimental crack pattern with the DIANA smeared cracking planes for the fully filled and partially filled panels (Fig 15). The stress field in the panels tends to force the fracture cracks to follow the line of least resistance rather than the line of action of the splitting load just like happen in the experimental tests. The results of this study indicate that the numerical FEM analyses were able to well describe both the trends and the variability of the four experimental tests.

4.2 Modelling with UDEC

Geometric models of the wall panels tested in the laboratory were created in UDEC. Tuff blocks were modelled as deformable block behaving according to UDEC's Mohr-Coulomb Plasticity model. Mortar joints were represented by interfaces behaving according to UDEC Coulomb slip model [21]. As well as in the case of FEM modelling the workmanship defects have been simulated by modelling an equivalent reduction of the width (out of plane) of the mortar joints. The mortar joints width considered are the same used in the FEM modelling and they are reported in Table 7. Some of the material parameters obtained from micro-scale experimental tests (Table 8) while other computed(Table 9). In particular, the elastic normal stiffness (JKn) has been computed as the ratio between the Young modulus, E, and the mortar joint thickness, t: JKn = E/t. The angle of friction (Jfric) has been computed as: Jfric = $(f_c-f_t)/(f_c+f_t)$, where f_c is the mortar compressive strength and f_t is equal to Jten. The cohesive strength (Jcoh) has been computed as Jcoh = $1/2\ (f_c f_t)^{1/2}$.

The boundary conditions assigned in the model were to represent the conditions of the laboratory test set up. Thus, the base has been fixed and the platen has been constrained to move only in the vertical direction. The model was brought initially at equilibrium. Then external loading has been applied. A constant vertical velocity was applied at the load spreader plate on the top of the wall panel. The velocity was converted to a vertical displacement and the force acting on the spreader plate for each load increment estimated. Hence, load versus displacement relationships were determined for the panel. Convergence tests were carried out on the magnitude of velocity to be applied to the spreader plate to make sure that a quasi-static loading condition was achieved. Fig. 16 compares the UDEC against the results obtained from the experiment. Fig. 16 shows the failure mode of the tuff masonry wall panel as predicted from UDEC. Also, Fig. 17 compares the load displacements curves obtained from UDEC against that from the experiment. The results predicted from UDEC are higher than the experimental results. This is as a result of the brittle nonlinear behaviour of the blocks which strongly influence and limit the performance of the wall panel. Furthermore, being the plasticity in the bricks modelled by means of constant-strain triangular elements (compared to the eight-node quadrilateral isoparametric plane stress element used in FEM). an overestimation in the failure load is expected [21]

CONCLUSIONS

An evaluation of the suitability of FEM and DEM approaches to analyse the behaviour of low strength masonry has been conducted. The approaches have been validated by means of two case-studies. In particular, numerical FEM and DEM outcomes and experimental results, for different low strength masonries have been compared. The main purpose of the current study was to give a contribution to the knowledge and selection of the more reliable approach to study this kind of structures. The analyses have shown that, for low bond strength masonry, where the emphasis is on joint behaviour, DEM approaches perform better. Since the bricks are highly stronger than the mortar, they could be even considered as rigid blocks. Moreover, the small displacement assumption could not be always satisfied and the rocking effect could be crucial. In these conditions the use of a refined plasticity model, for the bricks, became less significant, while a large displacement assumption could become necessary. Then the DEM approach is more reliable, in particular to predict the behaviour till failure, where new contacts could also form. However at the large scale, both DEM and FEM approaches are good to model the behaviour until the first crack though. In the case of low unit strength masonry, the FEM approach is the more reliable. In the considered case study, by means of the FEM modelling the experimental behaviour in terms of first crack, trend, failure and smeared crack pattern has been simulated. While the DEM model was not able to catch the experimental behaviour. In the case of the low unit strength masonry, indeed, a refined and reliable plasticity (and cracking model) for both the brick and the mortar, is crucial. In conclusion, despite the larger number of parameters required for the modelling, the FEM approach is a good choice for the low unit strength masonry. On the other hand, DEM is the preferable approach for the low bond strength masonry and, apparently, less parameters are needed for the modelling. It is not trivial to achieve these parameters, even performing specific tests. Therefore, often, optimization analysis is needed to obtain reliable mechanical parameters. Neither the FEM nor the DEM approach could be considered “reliable in every case”. At the micro scale, careful validation as well as an analysis of the influence of parameters and calibration of the model are always required.

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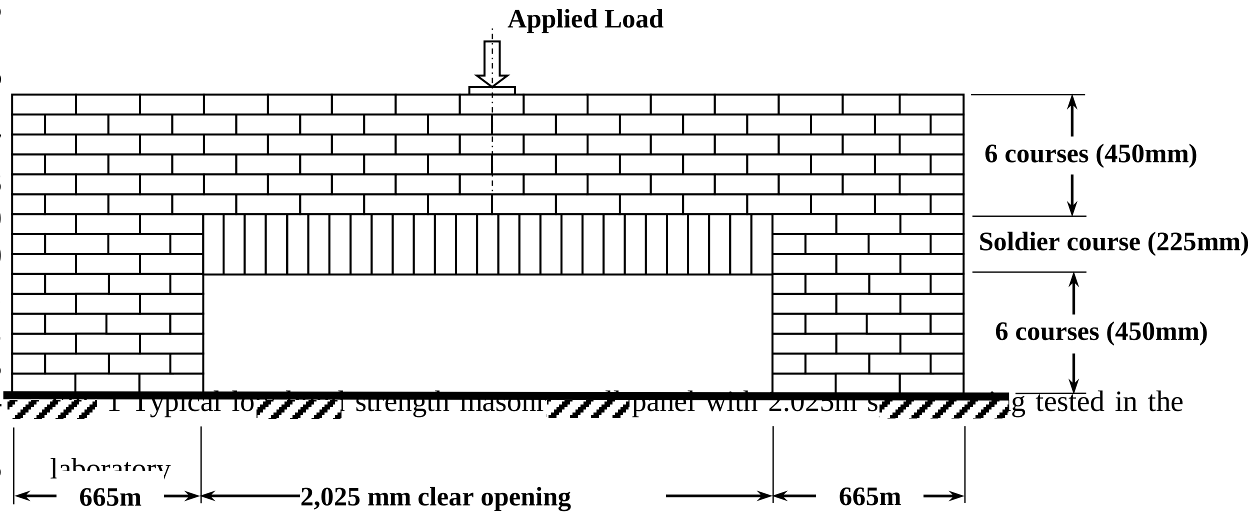


1 Typical lo[...] strength masonr[...] panel with 2.025m s[...]g tested in the laboratory

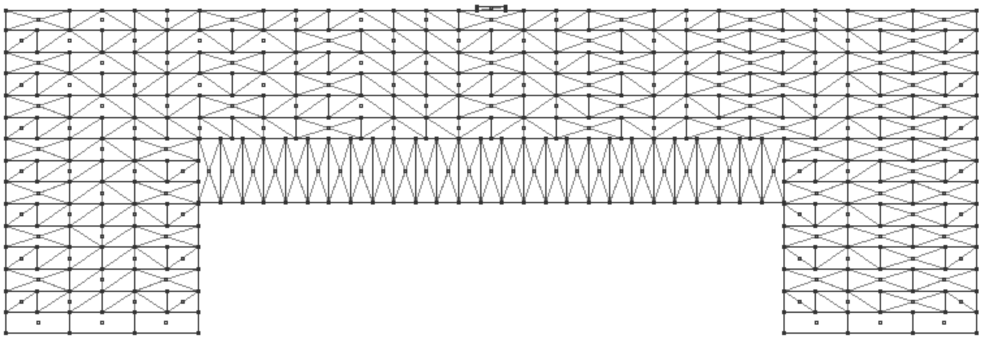


Fig. 2. UDEC geometric model of a masonry wall panel with a 2.025m opening

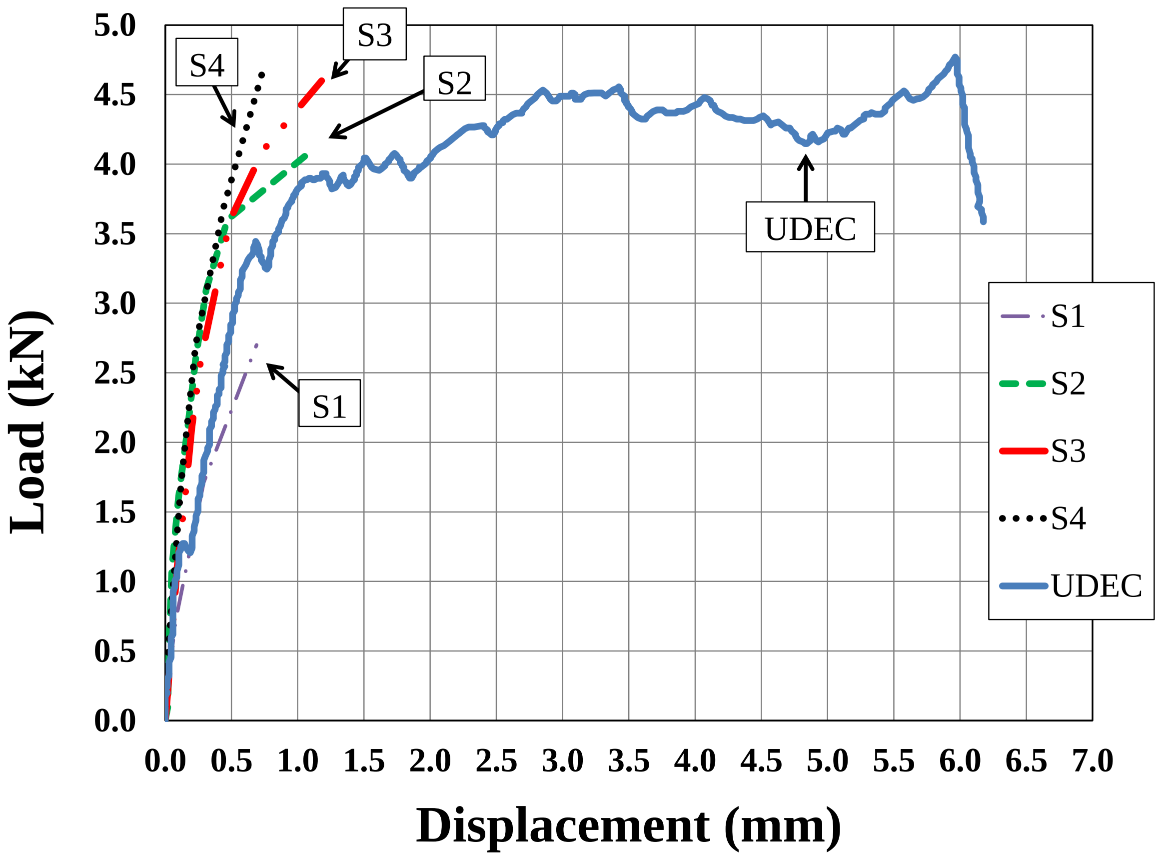


Fig. 3. Comparison of experimental against numerical results as obtained from UDEC

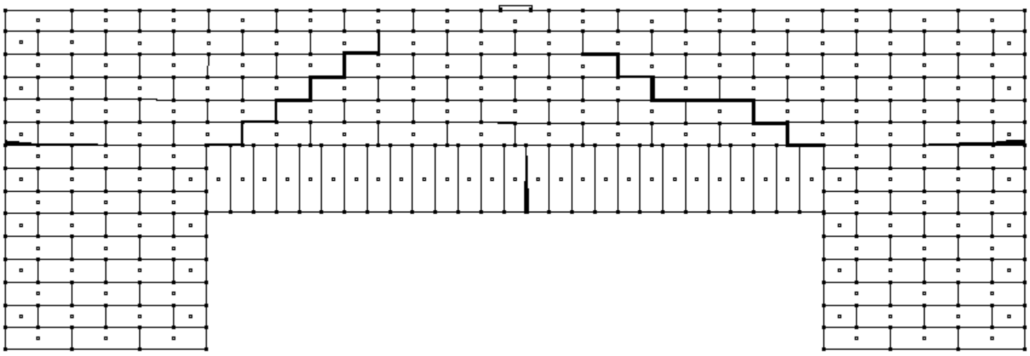


Fig. 4. Failure mode of the masonry wall panel as predicted with UDEC

Fig. 5 Failure mode of the masonry wall panel as observed from the experiment

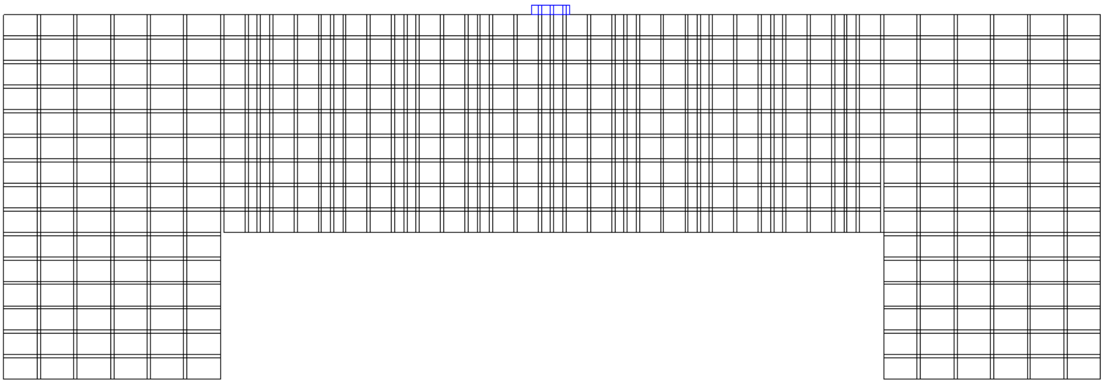


Fig. 6. Geometry of the model in DIANA

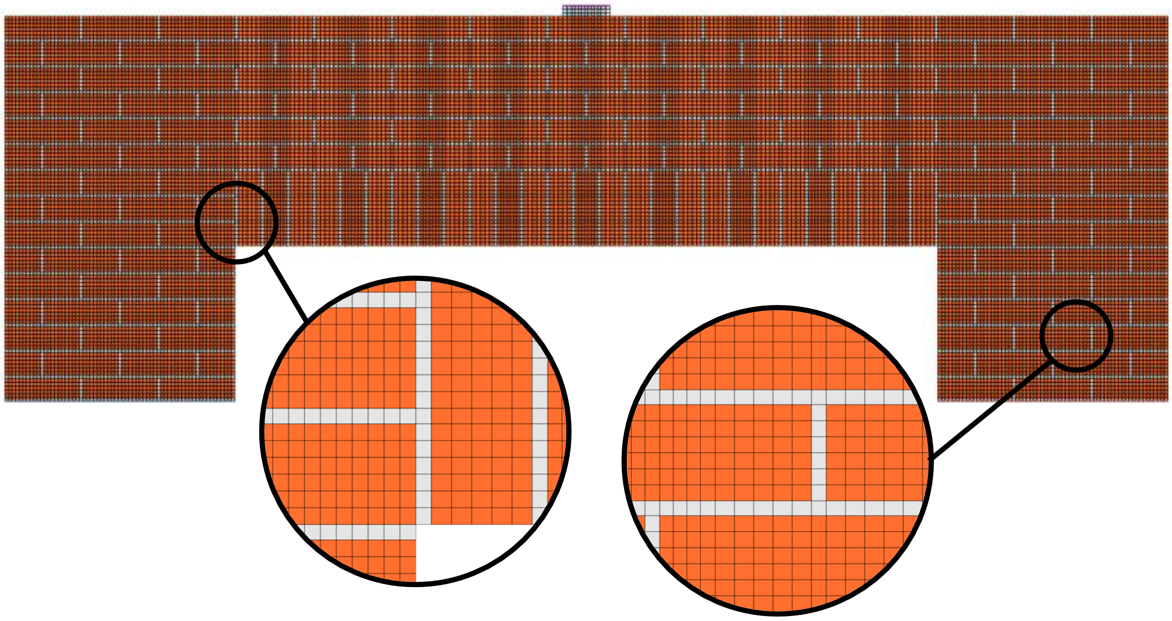


Fig. 7. Details of the adopted fine mesh for the DIANA FEM model of the wall panel (note that the colours are related to the materials)

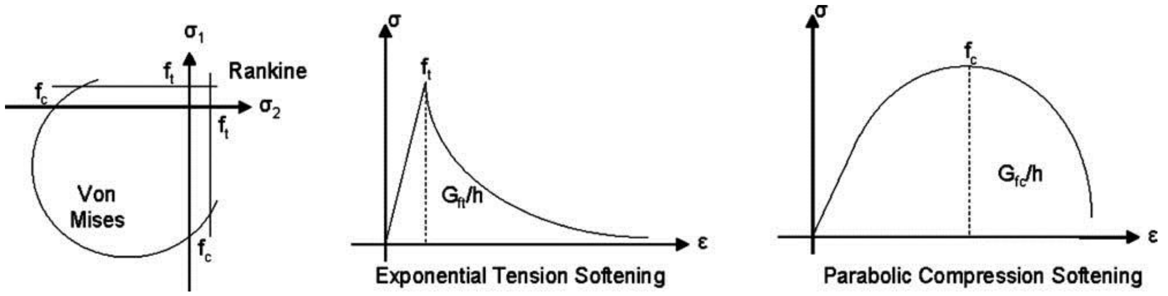


Figure 8. Material models used in DIANA

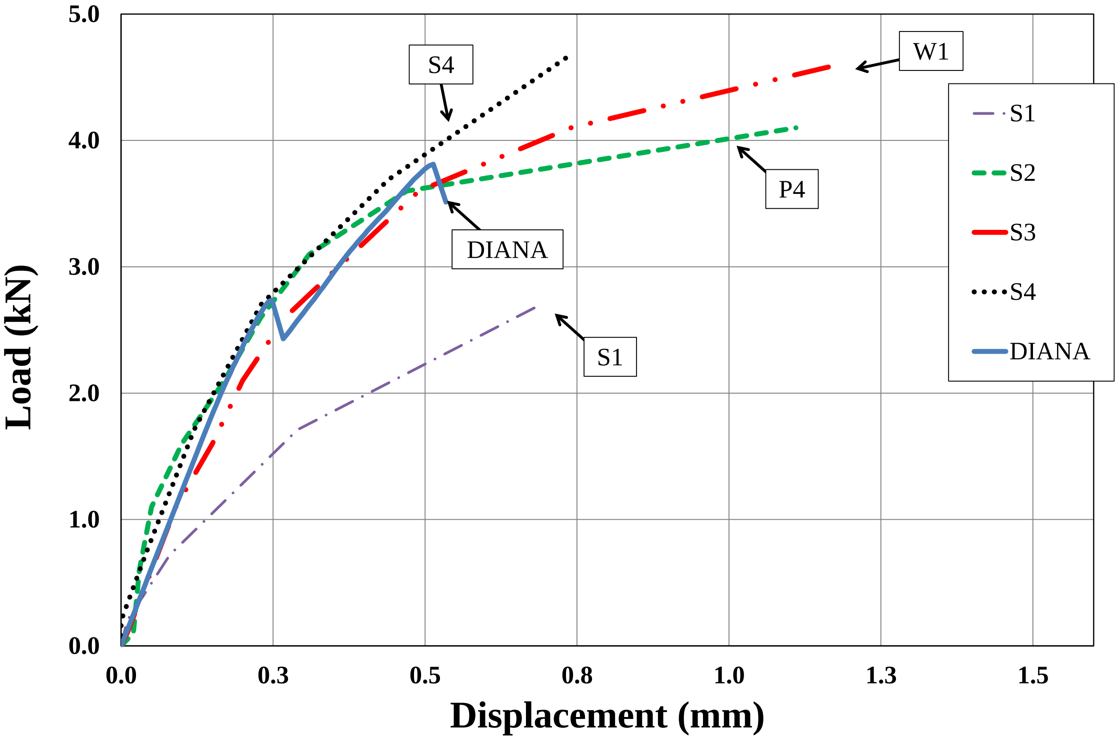


Figure 9. Comparison of experimental against numerical results as obtained from DIANA

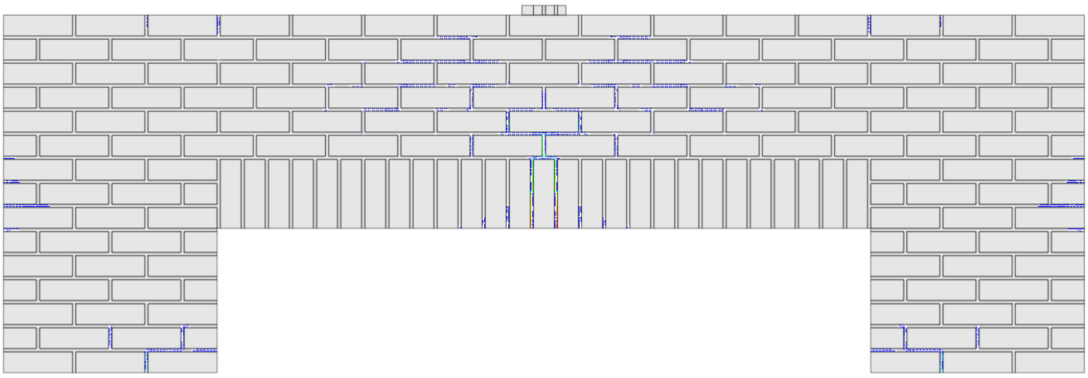
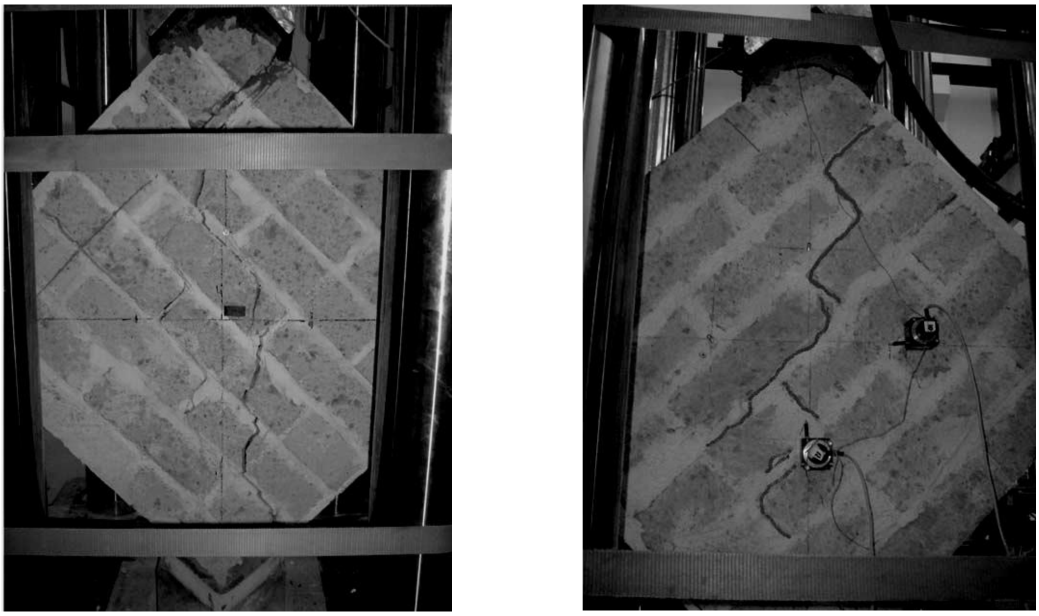


Fig. 10. Smeared Crack Pattern of the masonry wall panel as predicted with DIANA

Figure 11. Experimental setup for low unit strength masonry wall panels



a) Panel P2 b) Panel P4

Figure 12. Experimental crack pattern: a) Panel P2, b) Panel P4

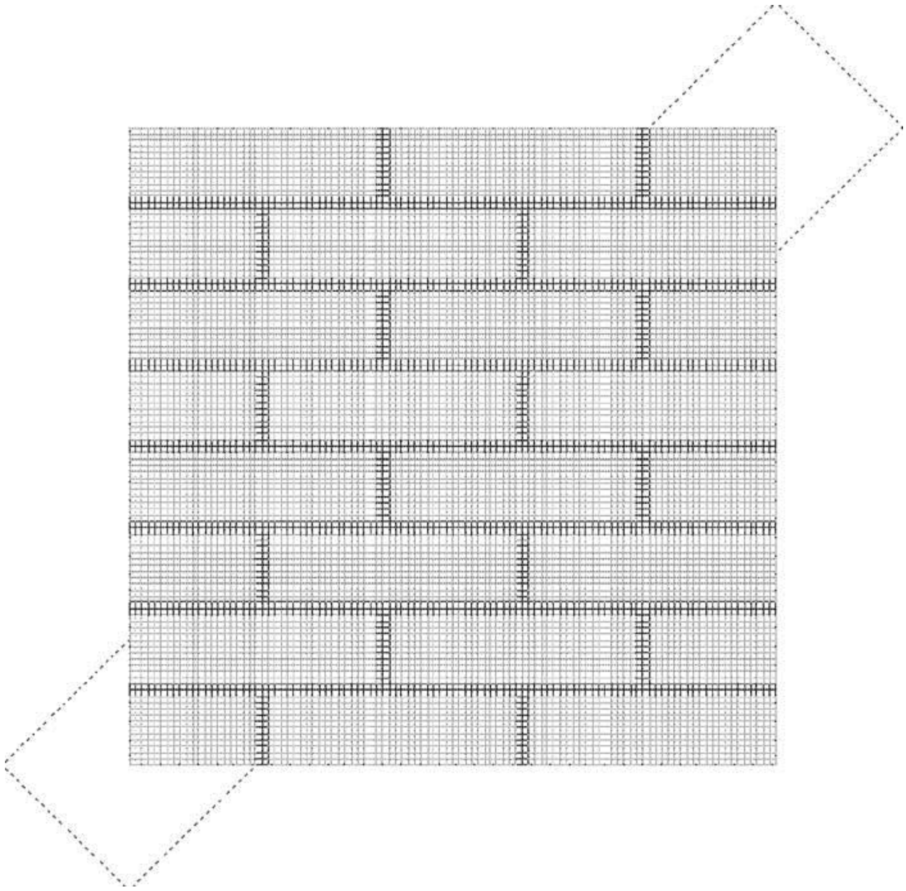


Figure 13. Mesh adopted for DIANA FEM modelling of the wall panel

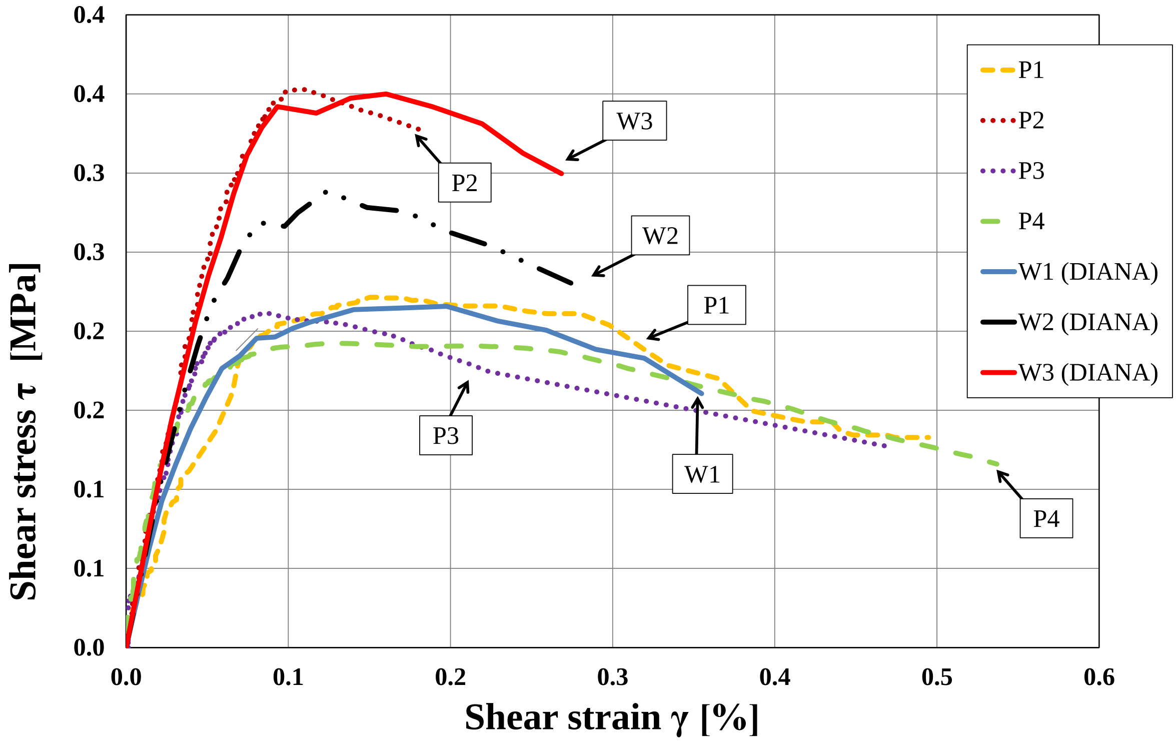
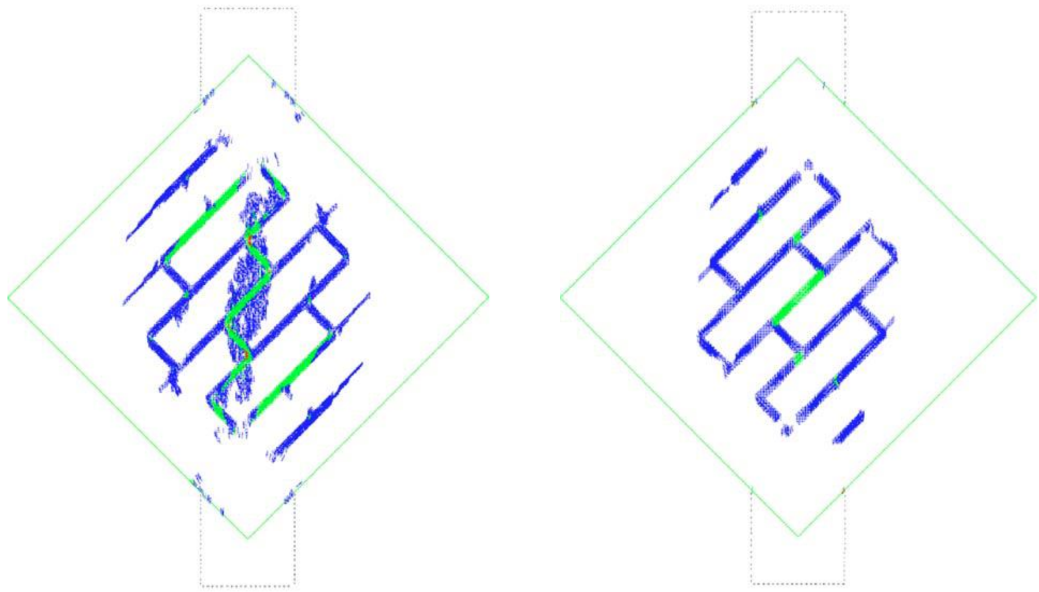


Figure 14. Comparison of experimental against numerical results as obtained from DIANA accounting for workmanship defects



a) Full joint (W3) b) Partial joint (W1, W2)

Figure 15. Smeared Crack Pattern of the masonry wall panel as predicted with DIANA

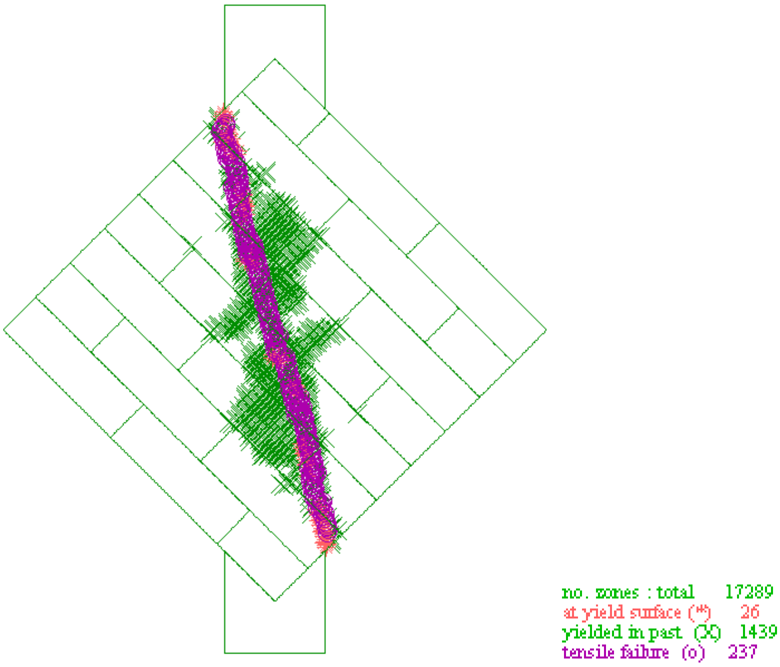


Figure 16. Failure mode of the masonry wall panel as predicted with UDEC

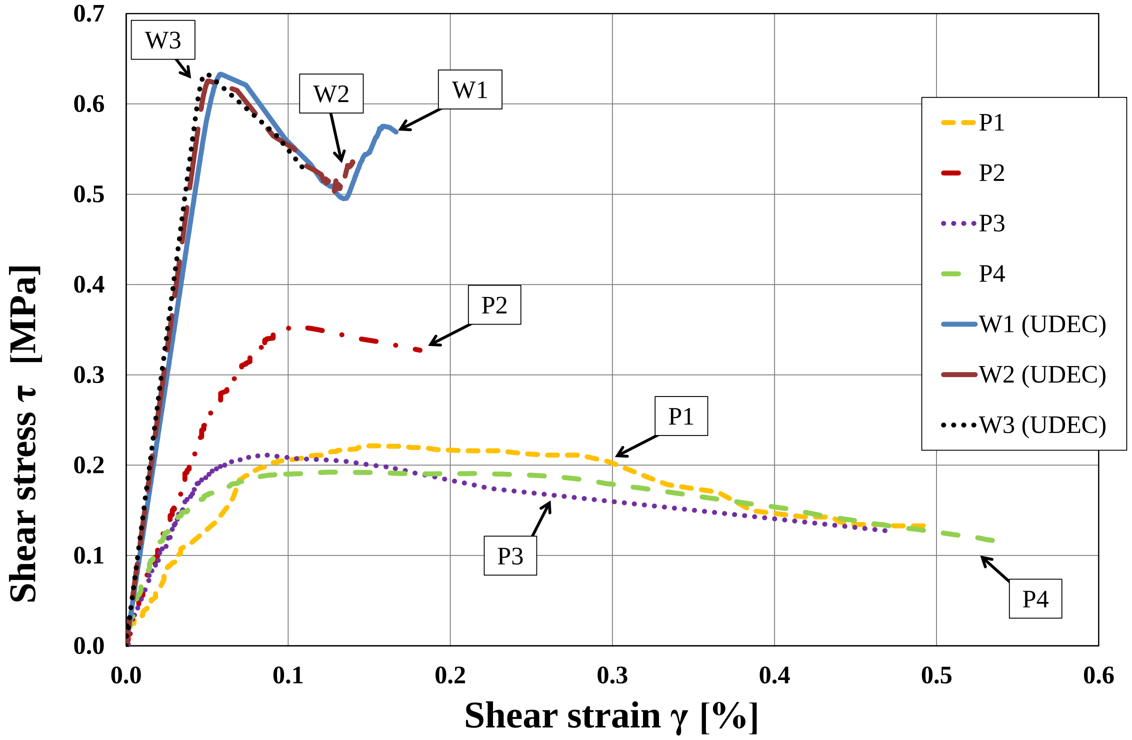


Figure 17. Comparison between experimental and numerical curves with UDEC